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Numerical Modeling System for Shoreline Change

by

Hans Hanson

OCTOBER 1986

United States Army

EUROPEAN RESEARCH OFFICE OF THE U.S. ARMY

London   England

CONTRACT NUMBER DAJA45-85-C-0032

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20. ABSTRACT *(Continue on reverse side if necessary and identify by block number)*

This report describes a combined system of numerical models called GENESIS, developed for calculating shoreline change as caused primarily by wave action. The system is based on the one-line theory, for which it is assumed that the beach profile remains unchanged, thereby allowing beach change to be described uniquely in terms of the shoreline position. As opposed to previous models based on the same concept, GENESIS is generalized in the sense that a simple user interface allows the system to be applied to a diverse variety of situations involving almost arbitrary numbers, locations, and combinations of groins, ——

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jetties, detached breakwaters, seawalls, and beach fills. Other features included in the system are wave shoaling, refraction, and diffraction; sand passing through and around groins, and sour s and sinks of sand. An overview of the modeling system is presented, and simulations of hypothetical as well as proto- type situations are given to demonstrate the capabilities of the system.

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PREFACE

The investigation presented in this report was conducted partly at the Coastal Engineering Research Center (CERC), Waterways Experiment Station (WES), Vicksburg, Mississippi, and partly at the Department of Water Resources Engineering, University of Lund, Institute of Science and Technology, Lund, Sweden.

1. INTRODUCTION

The acronym GENESIS stands for GENEralized model for SImulating Shoreline change. GENESIS is a combined system of numerical models which allows simulation of shoreline change occuring over a period of months to years, as caused primarily by wave action. The system is generalized in the sense that the model can be used to simulate shoreline change under a wide variety of user specified beach and coastal structure configurations. In addition, the input wave conditions can be entered from an arbitrary depth as a single value specified by the user (simlified wave refraction calculation, assuming parallel bottom contours), or through interaction with a more rigorous wave refraction model (allowing specification of an irregular bottom bathymetry).

GENESIS is based on the shoreline modeling work of the author and collaborators carried out over the past several years. Before the development of GENESIS, each application of a numerical shoreline model required extensive modification of an existing model and special refinements, as necessary, for the particular study. Considerable time was spent in altering the internal structure of the model, as well as on the data entry to arrive at a configuration which allowed easy modification in order to investigate design alternatives. With the experience gained in a variety of applications, the possibility became apparent of combining, in a general way, all major features of previous models into one shoreline modeling system. The remaining task would be to structure the system in such way that a general interface would allow the user to operate the model with minimum effort. In essence, the user would interact only with the interface and not with the model system itself.

With this goal, an 18-month effort was spent in developing the first version of GENESIS described herein. This effort was greatly advanced by a 6-month intensive work period hosted and supported by the Coastal Engineering Research Center (CERC) of the U. S. Army Engineer Waterways Experiment Station. During the course of this project, not only was the shoreline modeling system generalized, but many existing modeling concepts and algorithms were extended and refined. New developments were also included, and the system was tested against shoreline change data from both laboratory physical models and real beaches.

At present, through an easy to use interface, the model can be applied to simulate shoreline change including the effects of groins, jetties, detached breakwaters, seawalls, and beach fill. Almost arbitrary numbers, locations, and combinations of such structures can be represented. The model is economical to run and,

therefore, simulations can be performed for wide spatial extents and long time intevals.

Naturally, GENESIS has limitations in its use. This report must be carefully read to understand the limitations of the model, as well as to apply it correctly. Although GENESIS underwent a considerable period of development, much remains to be done. The present configuration of the model is only the first version in an anticipated series of model development steps, some of which are already in progress.

At present, the model is being used at CERC and at the University of Lund to investigate actual prototype applications concerning the forecast of future shoreline change, and for recommendations of remedial measures against cronic beach erosion. Copies of the program reside at the Department of Water Resources Engineering, University of Lund and at CERC.

2. BACKGROUND

Sandy beaches are in a continuous process of transformation in response to changes in the local waves and currents, and to variations in the amount of sand available for transport. It is a common observation that, although a beach may vary slightly in position and slope under seasonal changes in the wave climate, the profile of a particular beach oscillates about an apparent constant shape over the long term. This observation led Pelnard-Considere (1956) to develop a mathematical model, now called the one-line model, which was first numerically implemented by Price, Tomlinson, and Willis (1972) followed by others.

According to the one-line theory, erosion or accretion of a beach results in a pure translation of the beach profile (see Figure 1), that is the bottom profile moves in parallel to itself without changing shape. Hence, only one point on the profile is needed for describing the movement or location of the entire cross-section. As this point is conveniently taken to be the shoreline, the model is often called the shoreline model.

The one-line model was later modified to permit a schematic description of changes in the bottom profile. This was accomplished by treating two contour lines (Bakker 1968; Bakker, Klein-Breteler, and Roos 1970; Horikawa, Harikai, and Kraus 1979), and an arbitrary number of lines (Perlin and Dean 1978, 1983). However, due to a lack of understanding of the physical phenomena involved, in particular of the cross-shore transport rate, for which no reliable quantitative relation has yet been established, these multi-line models have not found much engineering use.

Numerous studies have been made with the one-line model to examine shoreline change in laboratory (physical) models (e.g. Pelnard-Considere 1956; Mimura, Shimizu, and Horikawa 1983; Perlin 1979) as well as under prototype conditions (Willis 1977; Sasaki, and Sakuramoto 1978). However, only Kraus, Hanson, and Harikai (1985) and Hanson and Kraus (1986) present an attempt to use the model as an engineering tool for making shoreline change forecasts for a real beach. Based upon the results of these latter two studies, recommendations for remedial measures were given.

The model presented by Kraus and Harikai (1983), Kraus et al. (1984), and Hanson and Kraus (1986), developed specifically to simulate conditions at Oarai Beach, Japan, was reformulated in a generalized form, leading to the modeling system GENESIS, making the model applicable to an arbitrary open-coast beach. This report describes the generalized model; how it functions and how it is initialized to take into account user-specified structures and

operations that can be introduced almost arbitrarily in space and time.

To assist in understanding the model and its capabilities, examples are given. The model is also applied to prototype conditions, reproducing shoreline change at an actual engineering project.

3.   GENERAL REMARKS

3.1.   One-line theory

In   response   to variations in incident wave characteristics, particularly to seasonal changes in wave height and   period,   sand moves   onshore and offshore, causing the nearshore bottom topogra-phy to change. This is most   clearly   demonstrated   during   severe storms,   which   can produce dramatic changes in the bottom profile shape and the shoreline position.   However, unless   the   storm   is unusually   severe,   a short time after it has passed, typically on the order of days or weeks,   the   beach   profile   returns   to   its pre-storm   shape   and   position   through   the   same   mechanisms of cross-shore transport.

In contrast, inbalances in the <u>longshore</u> sand   transport   rate causes   more   gradual and permanent changes in the beach planform. In this process the beach profile appears   to   remain   essentially unchanged, and the time-scale involved is normally on the order of months or years.

Based on the observation that the shape of the   beach   profile is relatively stable in a long-term perspective, Pelnard-Considere (1956) formulated what was later called the one-line   theory.   The basic   assumption   is that the bottom profile moves in parallel to itself (see Figure 1) without changing its shape during   the   pro-cess   of   erosion or accretion.   Hence, only one point on the pro-file   is   needed   for· determining   the   location   of   the   entire cross-section.   As   this   point   is   conveniently   taken to be the shoreline, the theory is usually called the shoreline theory.
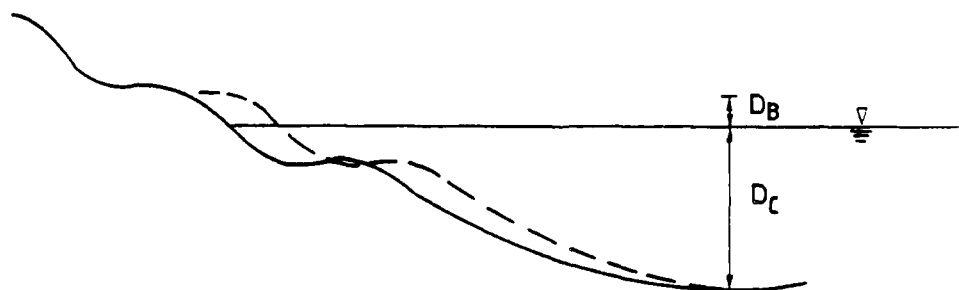
FIGURE 1.   Shoreline change and associated bottom profile.

In many applications, it is acceptable to ignore the short-term shoreline fluctuations in the beach profile and, instead, focus on the long-term changes. For these cases, the one-line model is a suitable simulation instrument. Moreover, a study of shoreline change at Oarai Beach, Japan, (Kraus and Harikai 1983) demonstrated that, in the neighborhood of the large structures at Oarai Harbor, short-term fluctuations of the shoreline position are smaller than those on the adjacent open coast. This result indicates that the one-line theory is particularly suited to reproduce shoreline evolution in the vicinity of coastal structures.

The same philosophy, that a specific beach has a characteristic profile depending mainly on the sediment properties and almost independent of the temporal variations in wave climate, is reflected in the "equilibrium beach profile" concept (Bruun 1954; Dean 1977). The equilibrium concept led to the development of the multi-line model, in which cross-shore sand transport can be characterized to some extent, as well as the cross-shore distribution of the longshore transport rate. Elementary forms of this model were presented by Bakker (1968) and by Bakker et al. (1970) in terms of analytic solutions, and by Perlin and Dean (1978), Horikawa et al. (1979), and Perlin and Dean (1983) by using numerical models. However, because the state of the art does not yet allow prediction of the cross-shore transport rate in terms of the local waves and currents, these multi-line models have not found extensive engineering use. In addition, multi-line models are much more costly to run, in terms of both required computer memory and execution time, as compared to a one-line model.

3.2. Problems addressed

When constructing or modifying a coastal facility, or when planning a non-structural coastal operation (e.g., a beach fill), it is important to be able to estimate the effects of the activity in terms of long-term accretion or erosion, along the beach and through the course of time. As examples, sediment borrow areas are preferrably located in coastal areas where the transport rate is high, to obtain quick recovery of the dredged hole, whereas a marina or small harbor would suffer the least shoaling and siltation problems if placed where the transport rate is low. A one-line simulation model can be of help to determine these locations and also to assess the resultant shoreline change.

As a secondary effect of nearshore activities, such as the construction of harbors, jetties, groins or revetments, the adja-

cent coastal area may suffer from a reduction in longshore sand transport. This, in turn, is reflected in a long-term shoreline change. In order to minimize these negative effects, simulations should be performed to estimate the course of shoreline change after the project is completed.

In situations where chronic beach erosion prevails, and the cause of erosion can not be remedied, engineers are often faced with the problem of having to protect the beach with some type of coastal structure or to renourish the beach. Often, these two types of measures are utilized in combination. For developing the design in terms of optimum location and properties for minimizing the beach changes, engineering know-how and past experience may not be sufficient. Here, numerical models can be used in the design process to guide and help quantify the conclusions drawn from coastal experience.

For cases where the sand transport capacity of the waves and currents remains essentially unchanged, the behavior of a beach fill can be estimated (James 1975; Shore Protection Manual (SPM) 1984). However, beach reclamation projects are often combined with the construction of structures which partially hold the sand, or which alter the local wave climate. In these cases, the design and maintenance of the fill are more complicated to predict. In situations like these, the one-line model can provide guidance.

4. BASIC MODEL PROPERTIES

4.1. Overview of limitations and assumptions

The fundamental assumption of the one-line model, that the bottom profile does not change in time, implies that only longshore sand transport can be taken into account. The longshore sand transport rate is assumed to have a uniform cross-shore distribution, and to be induced by the action of the breaking waves. As shown later, cross-shore transport can be simulated in a schematic way, in terms of non-wave induced sources and/or sinks along the coast (e.g., discharge from rivers, shoaling of harbors, removal of sediment by mining, etc.).

The second major assumption of the model is that sand actively moves over the profile to a certain limiting depth, beyond which the bottom does not move. This depth is called the depth of closure, D_C.

Although sand movement alongshore is assumed to be produced by waves and wave-induced currents, the details of the nearshore circulation usually are ignored. One exception is the circulation pattern in the shadow region behind structures, as caused by a strong gradient in wave height. This circulation is found to have a significant effect on shoreline evolution near structures (Kraus and Harikai 1983; Kraus 1983).

As explained in more detail below, for the wave and sand transport calculations in GENESIS, the bottom profile is assumed to follow the shape of the equilibrium beach profile (Dean 1977). One implication of this is that the depth increases monotonically. Thus, a particular point on the beach profile can be determined uniquely from the water depth, and a location at a greater water depth is always seaward of one at a lesser depth.

4.2. Fundamental relations

Following the assumption that the bottom profile moves in parallel to itself out to the depth of closure, continuity of sand for an infinitely small length, dx, of shoreline can be formulated as (Figure 2)

$$\frac{\partial y}{\partial t} + \left(\frac{\partial Q}{\partial x} + q\right) / (D_B + D_C) = 0 \qquad (1)$$

where y is the shoreline position, x is the longshore coordinate, t is the time, D_B is the average berm height above the mean water level, D_C is the depth of closure, Q is the longshore sand transport rate (m^3/s), and q represents line sources and/or sinks along the coast $(m^3/s/m$ shoreline) - positive for sources. In order to solve Eq. (1), expressions for the three quantities D_C, Q, and q must be formulated. The berm height, D_B, is taken from the measured or assumed profile.
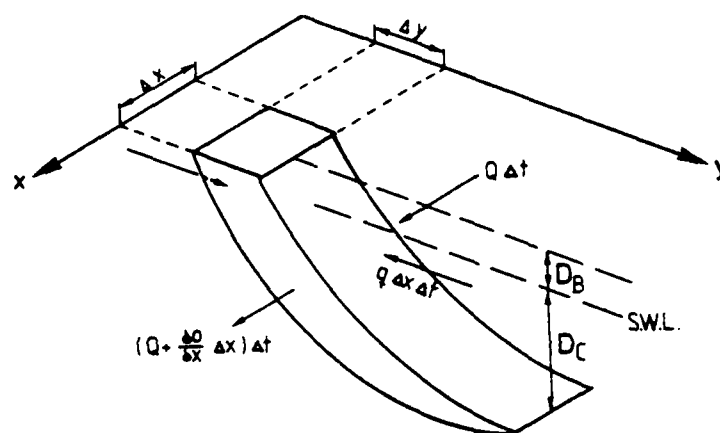
FIGURE 2.   Continuity equation interrelations.

4.2.1.   Depths of Sand Transport and Profile Change

For applications involving bypassing of sand at structures, knowledge of the depth to which sand is actively transported is required. This depth depends on the incident wave conditions which vary with time. This depth, assumed to be related to the incident wave height and period, is here called the depth of longshore transport, D_{LT}. Without cross-shore sand movement, the beach profile would change between this depth and the shoreline only, whereas other parts of the profile (above the shoreline and where the depth is greater than D_{LT}) would not move. However, cross-shore sand transport acts to smooth out the profile.

As previously mentioned, studies of real beach change taking place over a long period of time (years) indicate that the profile varies out to the depth of closure, D_C, associated with the wave

climate over this long time period. Various values have been suggested for this depth. Kraus and Harikai (1983) reports a value of 6 meters for Oarai Beach, Japan, where the maximum annual wave height is close to 3 meters. Other studies give similar results; twice the mean breaking wave height (Willis and Price 1975), twice the maximum breaking wave height (Sunamura and Horikawa 1977), approximately 1.3 times the breaker height (Walton and Chiu 1979), and twice the height of the 5-year return period wave height (Hands 1984). These are all of the same order as the formulation of Hallermeier (1983), giving the annual depth of closure as slightly more than twice the extreme annual significant wave height. In the light of these formulations, and keeping the potential errors involved in determining these relations in mind, GENESIS uses a simple relation for calculating the depth of closure as

$$D_C = 2 \, H_{mas} \tag{2}$$

where H_{mas} is the maximum annual significant wave height for the existing shore site. The value of H_{mas} for a given site must be specified by the user when operating GENESIS.

In GENESIS it is also assumed that the dry portion of the beach profile, from the shoreline to the berm crest, moves with the wet part of the profile while maintaining its shape. The berm crest height, D_B, is specified by the user in the input file.

To the author's knowledge, no reliable quantitative relation between the instantaneous wave climate and the depth of longshore transport has been reported in the literature. However, as the relation presented by Hallermeier (1983) appears to be very well justified by data, this relation is assumed to be valid also on a short-term time basis (hours). This makes possible the formulation of such a quantitative expression for the depth of lognshore sand movement according to

$$D_{LT} = 2.28 \, H_s - 10.9 \, \frac{H_s^2}{L} \tag{3}$$

where H_s is the significant wave height and L is the wave length, both calculated in deep water. The second term in this equation is typically one order of magnitude smaller than the first; the depth of longshore transport is thus approximately twice the significant wave height in deep water.

Although the longshore transport is assumed to take place over a limited portion of the active beach profile, the on-offshore

water particle velocity under waves, d mean wave-induced and tidal cross-shore currents as well as wind-blown sand transport on the dry beach cause the beach profile to move, while its shape remains relatively unchanged from the berm crest to the depth of closure.

From the above discussion, it is seen that whereas the depth of longshore sand transport, D_{LT}, determines the amount of sand bypassing groins, the calculated shoreline change is related to the depth of closure, D_C.

4.2.2. Longshore transport

The longshore sand transport volume rate, Q, is calculated using

$$Q = (H^2 C_g)_b \ (a_1 \sin 2\alpha_{bs} - a_2 \cos \alpha_{bs} \frac{\partial H}{\partial x})_b \qquad (4)$$

where C_g is the group velocity (m/s), α_{bs} is the angle of wave crests to the shoreline, and the subscript b denotes the breaking condition.

The non-dimensional parameters a_1 and a_2 are given by

$$a_1 = K_1 \ / \ (16 \ (\rho_s/\rho - 1) \ (1 - p) \ 1.416^{5/2}) \qquad (5a)$$

$$a_2 = K_2 \cot \beta \ / \ (8 \ (\rho_s/\rho - 1) \ (1 - p) \ 1.416^{5/2}) \qquad (5b)$$

where ρ_s and ρ are the densities of the sediment (quartz sand) and water, p is the sediment porosity, and $\tan\beta$ is the average bottom slope from the shoreline to the depth of longshore transport, D_{LT}. The factor 1.416 is used to convert from significant to RMS wave height. The calibration parameters K_1 and K_2 determine not only the relative strength between the two terms, but also the time scale in the model.

The first term in Eq. (4) expresses the longshore transport rate due to obliquely incident waves, and is commonly known as the CERC-formula (SPM 1984). The second term, introduced by Ozasa and Brampton (1980), accounts for the longshore sand transport rate

caused by the longshore variation in breaking wave height. It is particularly effective in the diffraction shadow zone near structures. The importance of this term was emphasized by Kraus, Harikai, and Kubota (1981) and Kraus and Harikai (1983).

4.2.3. Bottom profile

The continuity equation (Eq. 1) does not require specification of the shape of the bottom profile, since it was derived under the assumption that the profile moves in parallel to itself. However, in order to calculate the average nearshore bottom slope, $\tan\beta$, to be used in the transport equation (Eq. 4), as well as for determining the location of the breaking waves, a shape of the profile is needed.

According to Dean (1977), the shape of the bottom profile can be expressed as

$$D = Ay^{2/3} \tag{6}$$

where D is the water depth (m), A is a scaling parameter ($m^{1/3}$), and y is the distance from the shoreline (m). Moore (1982) has given an empirically determined curve for A as a function of grain size. Equation 6 and Moore's results are used by GENESIS. The average nearshore slope, $\tan\beta$, for an equilibrium profile is defined as

$$\tan \beta = \overline{\frac{\partial D}{\partial y}} = \frac{1}{y_L} \int_0^{y_L} \frac{\partial D}{\partial y} \, dy = Ay_L^{-1/3} \tag{7}$$

where y_L is the width of the littoral zone and the overbar indicates an average value. According to Eq. (6),

$$y = (\frac{D}{A})^{3/2} \tag{8}$$

The width of the littoral zone, y_L, in which the entire longshore transport is assumed to take place, is defined according to

$$y_L \equiv (\frac{D_{LT}}{A})^{3/2} \qquad (9)$$

thus encompassing the equilibrium beach profile from the shoreline out to the depth of longshore transport, D_{LT}. This leads to

$$\tan \beta = (A^3 / D_{LT})^{1/2} \qquad (10)$$

The use of D_{LT} in Eq. (9) is consistent with the original development of this closure depth as defining the extent of significant littoral transport (Hallermeier 1983).

4.2.4. Sources and sinks of sediment

GENESIS, as a one-line model, cannot describe shoreline change produced by cross-shore transport as caused, e.g., by a change in wave steepness. Thus, the model can be used for predicting long term changes as caused by an imbalance in the longshore sand transport entering and leaving a particular portion of a beach, whereas it is not capable of reproducing the rapid short-term changes, such as bar formation, during and following a storm event.

Sediment discharge from rivers or losses due to harbor siltation can be treated in the model (Kraus and Harikai 1983; Hanson and Kraus, in prep.). In GENESIS, such contributions have to be explicitly specified by the user and included in the cross-shore transport term, q. Thus, the gain or loss of sediment will be constant in time and independent of variations in the wave climate.

4.2.5. Wave refraction

Local wave refraction from the input (reference) water depth to the breaker line is simulated using Snell's Law:

$$k_1 \sin \alpha_1 = k_2 \sin \alpha_2 \qquad (11)$$

where k is the wave number ($2\pi/L$), α is the local wave angle to

the bottom contour, and the index 1 (2) refers to the depth at point 1 (2). This relation assumes that the bottom contours are straight within each calculation grid cell. From this, the refraction coefficient, K_R, is given by

$$K_R = (\cos \alpha_1 / \cos \alpha_2)^{1/2} \qquad (12)$$

4.2.6. Wave diffraction behind structures

As demonstrated by Goda, Takayama, and Suzuki (1978), the directional spreading of real wind-generated waves results in a smaller diffraction coefficient than predicted by monochromatic wave diffraction theory. GENESIS therefore uses the simplified diffraction calculation procedure for waves with directional spread presented by Goda et al. (1978) and Goda (1984) to represent diffraction at structures such as detached breakwaters and jetties.

According to this procedure, the diffraction coefficient, K_D, obtained from Figure 3, where P_E is the relative wave energy along a line making the angle θ to the incident wave direction at the tip of the diffracting structure, and S_{max} is a wave concentration parameter. A lower value of S_{max} in Figure 3 corresponds to irregular wind waves, whereas a higher value corresponds to swell. The area where the angle θ is negative is called the shadow region, taking the diffraction of light as an analogy. Consequently, the area where the angle is positive is called the illuminated region. Kraus (1984) gives a step by step explanation of the calculation procedure.

4.2.7. Breaking criterion

In the model, a standard empirical depth-controlled breaker criterion is used:

$$H_b = 0.78 \, D_b \qquad (13)$$

where H_b is the breaking wave height and D_b is the corresponding

depth. Outside a region influenced by diffraction, the wave height is given by

$$H = H_{rf} \, K_R \, (\alpha_{rf}, D_{rf}, D) \, K_S(D) \, / \, K_S(D_{rf}) \tag{14}$$

where H_{rf} is the wave height at the input (reference) depth, D_{rf}, K_R is the refraction coefficient, K_S is the shoaling coefficient, D is the local water depth given by Eq. 5. The ratio $H_{rf}/K_S(D_{rf})$ represents the wave height in deep water if the wave is not refracted (usually denoted by H_O'). The wave height, H, is then compared with the possible breaking wave height at the particular depth. If breaking conditions are not reached, the calculation moves to a point closer to shore, until the breaking criterion is satisfied.

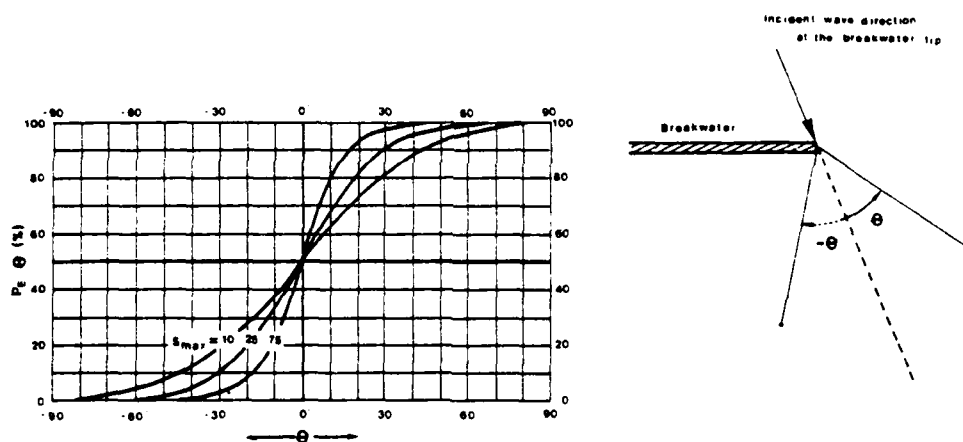FIGURE 3a. Wave energy cumulative   3b. Definition sketch
        curves.   for angle θ.
                    (After Goda, 1977)

In a shadow region, the wave height is give by

$$H = H_{tp} \, K_D(\theta) \, K_R(\alpha_{tp}, D_{tp}, D) \, K_S(D) \, / \, K_S(D_{tp}) \tag{15}$$

where K_D is the diffraction coefficient, θ is the geometric angle
for a line from the diffracting tip to the point considered,
measured relative to the wave direction at the diffracting tip
(see Figure 3b). The subscript, tp, refers to conditions at the
diffraction tip.

4.3. Wave energy windows

A central concept used in GENESIS, and one which determines the
algorithmic structure of the model, is that of __wave energy__
__windows__. This concept is simple, but provides a powerful and
general means to describe a variety of structural configurations
and physical conditions. An understanding of energy windows is
required in order to operate GENESIS.

An energy window is defined as an area open to the incident
waves. Windows are separated by a structure, such as a detached
breakwater or a diffracting groin. Incident wave energy must
enter through one of these windows to reach a location in the
nearshore area. As an example, in Figure 4, the energy windows
are labelled E1 - E5 and the structures S1 - S5. Although the
figure speaks for itself, a few comments shall be made.
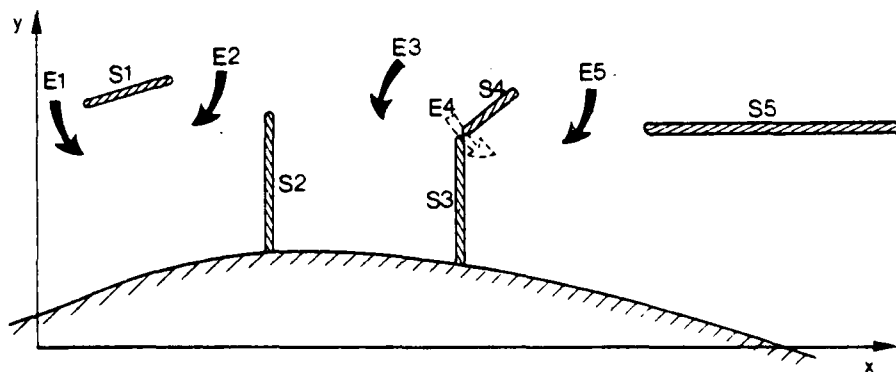
FIGURE 4.   Wave energy windows E_1 - E_5 and
            structural elements S_1 - S_5.

E1: This semi-infinite window is bounded only on the right-hand
    side. Thus, waves entering through this window are diffrac-
    ted by one structure, S1. Also, note that waves entering
    through this window cannot continue into the areas to the
    right of structures S2 - S4, and therefore do not affect the
    sand transport in those areas.

E2: This window is bounded on both sides and the waves are, therefore, diffracted by two structures, S1 and S2. Waves entering through this window cannot reach the shoreline to the right of structures S2 - S4.

S2: Groins extending through the breaker zone are considered to influence wave breaking through diffraction. Also, as structures at this stage of the model development are not assumed to transmit wave energy, wave energy entering on one side of a structure cannot propagate to the other side of the structure. Waves entering through window E3 only reach the shoreline between S2 and S3.

S3 and S4: In GENESIS, the two basic diffracting structure elements, the groin and the detached breakwater, can be combined to create T-groins, dog-eared groins, etc. Between two such elements, no wave energy can pass. For this reason, the wave energy window E4 is said to be "closed" between S3 and S4.

S5: If the wave energy entering to the project area from the right side of this structure can be neglected, the structure can be assumed to be infinitly long, and shoreline change to the right of S3 and S4 governed solely by wave energy arriving through window E5.

5. FINITE DIFFERENCE REPRESENTATION

The system of basic equations (Eqs. 1 - 4) is now closed, making possible calculation of shoreline change. Analytical solutions can be be obtained for certain simplified conditions (Bakker and Edelman 1964; Bakker 1968; Le Mehaute and Soldate 1977; Walton and Chiu 1979; Larson, Hanson, and Kraus, in prep.), but in order to describe realistic situations involving a general shoreline, together with time-varying wave conditions, Eq. (1) must be solved numerically (e.g., Price et al. 1973; Sasaki and Sakuramuto 1978; Perlin 1978, 1979; Hanson and Kraus 1980; Harikai and Kraus 1983; Kraus 1983).

In the following notation, a subscript (i) is used to denote the location along the beach. A primed (') value refers to the new time level, and an unprimed value to the old.

By using finite differences, the shoreline is discretized on a staggered mesh, with the calculation grid points y_i and Q_i defined alternately as shown in Figure 5. For an arbitrary calculation element (c.f. Figure 2), the change of shoreline position, Δy, over a time increment, Δt, can be expressed as (c.f. Eq. 1)
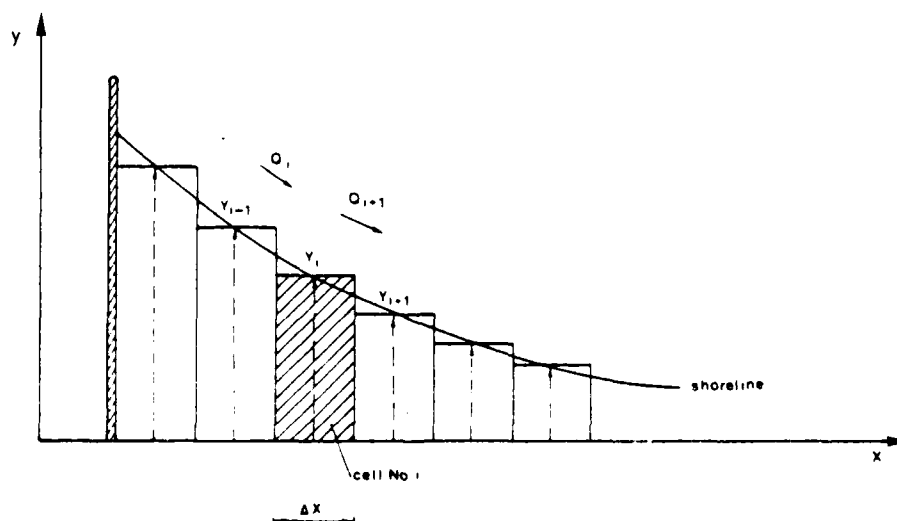
FIGURE 5.   Shoreline grid representation and associated longshore transport rates.

$$\Delta y = (Q_{IN} - Q_{OUT}) \frac{\Delta t}{\Delta x \cdot D} \tag{16}$$

where

$$Q_{IN} = Q + q \ \Delta x \tag{17a}$$

$$Q_{OUT} = Q + \frac{\partial Q}{\partial x} \ \Delta x \tag{17b}$$

$$D = D_B + D_C \tag{17c}$$

and Q is the volume rate of longshore sand transport (m^3/s), q is the volume rate supply (positive) or loss (negative) ($m^3/s/m$ shoreline), and Δx is the finite grid length alongshore.

In the model, y_i and Q_i are calculated along the shoreline alternately in time. As an initial condition, the position of the shoreline, y_i, is given at the center of each grid element. With this information and known breaking wave parameters, the longshore transport rate, Q_i, can be calculated for each grid point. The shoreline change, Δy_i, can then be calculated according to Eq. (16), giving the new shoreline location, y_i, along the beach.

5.1. Calculation of longshore transport

For convenience, Eq. (4) is reproduced here:

$$Q = (H^2 C_g)_b \ (a_1 \ \sin 2\alpha_{bs} - a_2 \ \cos \alpha_{bs} \ \frac{\partial H}{\partial x})_b \tag{4}$$

This relation is used for calculating the longshore sand transport rate, Q_i, for each calculation grid cell. The wave angle to the shoreline, α_{bs}, is defined according to Figure 6, and calculated as

$$\alpha_{bs} = \alpha_b - \alpha_s = \alpha_b - \tan^{-1} \frac{\partial y}{\partial x} \tag{18}$$
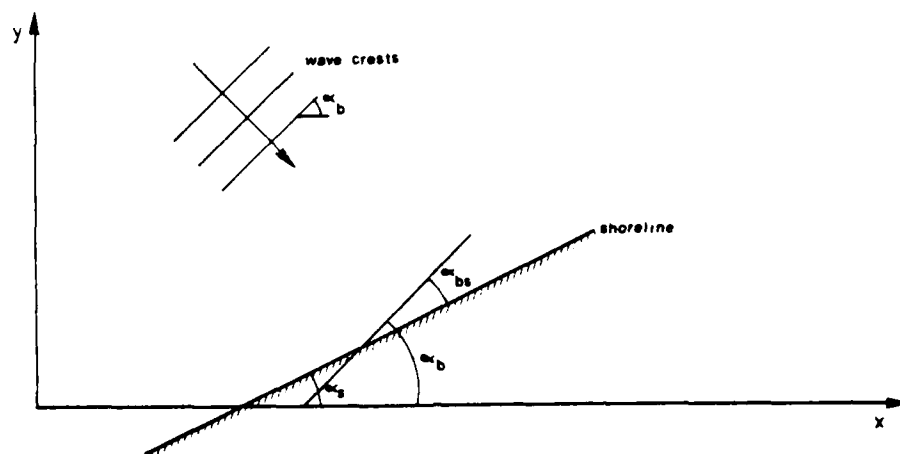
FIGURE 6.   Definition of breaking wave angles.

where α_b is the wave angle to the x-axis and α_s is the shoreline
angle to the x-axis. For the value of the longshore sand transport
parameter, K_1 in Eq. (5a), Komar and Inman (1979) and the Shore
Protection Manual (SPM 1984) recommend a value corresponding to
0.77, if the root-mean-square wave height is used.   However, this
value is heavily weighted by results from short-term tracer
experiments, and may not be representative for the long-term
shoreline evolution as simulated by the one-line model.   In addi-
tion, various approximations in the model may result in somewhat
different values than the true value for the particular coast.
Previous shoreline model studies (e.g., Kraus and Harikai, 1983;
Kraus et al. 1984) suggest values of K_1 in the range of 0.1 to
0.5.

     The other longshore transport parameter, K_2 in Eq. (5b), has
been assigned a wide range of values (Gourlay 1982).   The studies
of Kraus and Harikai (1983) and Kraus (1983) indicate that the
ratio K_1/K_2 for prototype applications is on the order of 0.5 to
1.5. For numerical simulation of physical scale model tests, Kaji-
ma et al. (1983) suggest a value for this ratio of 0.88, based on
a hydrodynamic laboratory model study of the nearshore circulation
near a structure. Figure 7 demonstrates the influence on various
values on K_1 and K_2 on the shoreline evolution behind a detached
breakwater.   As demonstrated, the first term in Eq. (4) tends to
flatten out the salient (evolving tombolo) behind the detached
breakwater, whereas the second term tends to promote growth of the
salient.
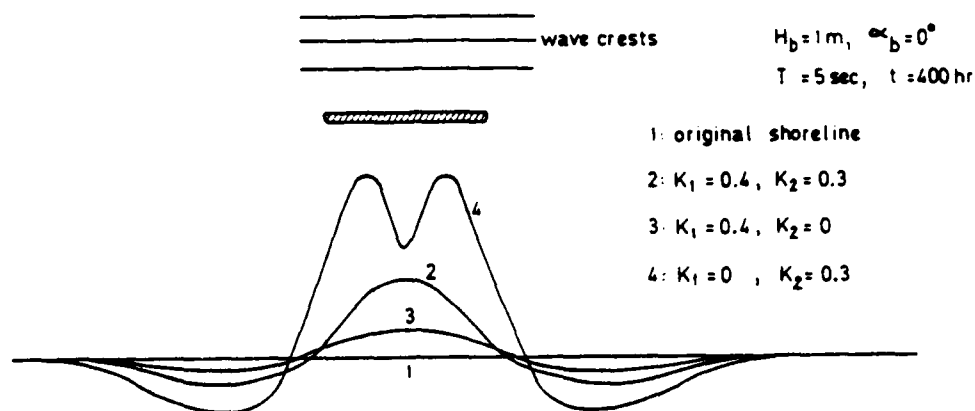
FIGURE 7. Demonstration example showing the influence of the two terms in the sand transport equation. $H_b = 1$ m, $\alpha_b = 0$ deg, $T = 3.5$ s. Simulation time = 90 days.

5.2. Stability criterion

In applications of a one-line numerical model with an explicit-type solution scheme (see below), it is found that for certain combinations of Δt and Δx, the solution becomes unstable. This condition manifests itself by causing increasingly larger jagged oscillations in the shoreline position. By linearizing Eqs. (1) and (18) with respect to y, it is possible to derive a stability parameter, the value of which can be used for determining the stability of the calculation scheme. This will now be shown.

Assuming small breaking wave angles, Eq. (18) can be written

$$\alpha_{bs} = \alpha_b - \frac{\partial y}{\partial x} \qquad (19)$$

Introducing Eq. (19) into the longshore transport relation leads to

$$\frac{\partial Q}{\partial x} = - (H^2 \, Cg)_b \; (2a_1 + a_2 \sin \alpha_{bs} \; \frac{\partial H}{\partial x})_b \; \frac{\partial^2 y}{\partial x^2} \qquad (20)$$

Assuming negligible contribution from sediment sources and sinks ($\partial q/\partial x = 0$), Eq. (1) becomes

$$\frac{\partial Q}{\partial x} = - (D_B + D_C) \frac{\partial y}{\partial t} \tag{21}$$

Combining Eqs. (20) and (21) yields

$$\frac{\partial y}{\partial t} = (\epsilon_1 + \epsilon_2) \frac{\partial^2 y}{\partial x^2} \tag{22}$$

where

$$\epsilon_1 = 2 (H^2 Cg)_b K_1 / (D_B + D_C) \tag{23a}$$

$$\epsilon_2 = (H^2 Cg)_b K_2 (\sin \alpha_{bs} \frac{\partial H}{\partial x})_b / (D_B + D_C) \tag{23b}$$

As Eq. (22) is a diffusion-type of equation, the stability of the explicit calculation scheme is well known:

$$\frac{\Delta t (\epsilon_1 + \epsilon_2)}{\Delta x^2} \leq 0.5 \tag{24}$$

This condition, called the Courant Condition in general, has to be met at every calculation grid point along the shoreline. The value of the expression on the left-hand side of the equation is called the stability ratio herein.

Although Eq. (24) was derived assuming small incident wave angles, tests with quite large breaking wave angles (< 45 deg), have shown that the condition expressed by Eq. (24) is still applicable.

5.3. Explicit Solution Scheme

The most straightforward approach to solve Eq. (1) is to use the relation

$$\frac{\partial Q_i}{\partial x} = \frac{Q_{i+1} - Q_i}{\Delta x} \qquad (25)$$

which, combined with Eq. (16), leads to the relation

$$y_i' = 2B (Q_i - Q_{i+1} + q_i \Delta x) + y_i \qquad (26)$$

where

$$B = \Delta t / (2 D \Delta x) \qquad (27)$$

The new shoreline position, y_i', is seen to depend only on values calculated at the previous time step. The main advantages of the explicit scheme are: easy programming, simple expression of boundary conditions, and shorter computer run time as compared with the implicit scheme (for a single time increment). A major disadvantage is, however, the stability of the solution, imposing a severe constraint on the longest possible calculation time step for given values on model constants and parameters. In practical applications, the time interval between given wave data may be too long to allow direct use of the explicit scheme.

5.4. Implicit Solution Scheme

If, the longshore transport rate derivative, $\partial Q_i / \partial x$, over a cell is expressed in the form

$$\frac{\partial Q_i}{\partial x} = \frac{1}{2} \left(\frac{Q_{i+1}' - Q_i'}{\Delta x} + \frac{Q_{i+1} - Q_i}{\Delta x} \right) \qquad (28)$$

we obtain

$$y_i' = B' (Q_i' - Q_{i+1}') + yc_i \qquad (29)$$

where

$$yc_1 = B (Q_i - Q_{i+1}) + B \Delta x (q_i' + q_i) + y_i \qquad (30)$$

The quantity yc_i can be interpreted as the shoreline position halfway between y_i and y_1'. It can be evaluated as it only depends on values at the old time step, and data.

The main advantage of the implicit scheme is that it is stable for almost any value of the stability ratio (cf. Eq. 24), although the computed result becomes increasingly inaccurate with larger stability ratios. As was shown in Hanson and Kraus (1986) for a specific case with a seawall, if the implicit calculation scheme was run with a time step four times longer than the longest possible time step for the explicit scheme, the total execution time was reduced to about 2/3 of that of the explicit scheme. The difference between the results for the explicit and implicit schemes was less than 1 percent for the particular example studied.

The main disadvantage of the implicit scheme is that the program, boundary conditions, and constraints become more complex, as compared to the explicit scheme.

5.5. Double sweep algorithm

As shown by Le Mehaute and Soldate (1978), Eq. (29) can be solved by using an iterative procedure. A more direct and computationally faster method is, derived through linearization of Eq. (4) with respect to y. Using the linearization method given by Perlin and Dean (1978) and extended by Kraus and Harikai (1983), the longshore transport rate, Q_i', at the new time step can be expressed as

$$Q_i' = E_i' (y_{i-1}' - y_i') + F_i' \qquad (31)$$

where E_i' and F_i' are functions of the incident wave parameters. Equations (29) and (31) form a tri-diagonal set of simultaneous equations for the y_1' and Q_i'. This system can be solved by using the so-called double sweep algorithm. This leads to an expression according to

$$Q_i' = EE_i' \, Q_{i+1}' + FF_i' \tag{32}$$

where

$$EE_i' = B_i' \, / \, (1 + B_i' \, (2 - EE_{i-1}')) \tag{33a}$$

$$FF_i' = \frac{F_i' + E_i' \, (yc_{i-1} - yc_i) + B_i' \, FF_{i-1}'}{1 + B_i' \, (2 - EE_{i-1}')} \tag{33b}$$

$$B_i' = B \, E_i' \tag{33c}$$

Equations (32) to (33b) are recurrence relations; the Q'-values are calculated in descending order, and the EE_i'- and FF_i'-values are calculated in ascending order. Therefore, the solution procedure becomes:

a. Specify the boundary conditions for Q_1' in terms of EE_1' and FF_1'.

b. Solve Eqs. (33a) and (33b) for i = 2 to N, where N is the total number of calculation cells. This is the first sweep through the grid.

c. Specify the boundary condition for Q_{N+1}' in terms of EE_N' and FF_N'.

d. Solve Eq. (32) for i = N to 1, thus performing the second sweep through the grid.

e. Calculate the new shoreline positions, y_i', according to Eq. (27).

As shown in Hanson and Kraus (1986), due to the seawall boundary condition, it is necessary to introduce another set of recurrence equations, similar to Eqs. (32) to (33c), but solved in the opposite direction according to

$$Q_i' = PP_i' \, Q_{i-1}' + RR_i' \tag{34}$$

where PP_i' and RR_i' depend on PP_{i+1}' and RR_{i+1}' repectively, and are defined similarly to EE_i' and FF_i', i.e.,

$$PP'_i = B'_i / (1 + B'_i (2 - PP'_{i+1}))$$ (35a)

$$RR'_i = \frac{F'_i + E'_i (yc_{i-1} - yc_i) + B'_i RR_{i+1}}{1 + B'_i (2 - PP'_{i+1})}$$ (35b)

5.6. Boundary conditions and constraints

Before any calculation of the longshore transport rate can start, boundary conditions must be formulated. The importance of boundary conditions cannot be overestimated, as all following calculations depend upon their formulation. In the one-line model, the boundary conditions are typically expressed in terms of the longshore transport rate at both ends of the calculation grid, although it is possible to express boundary conditions in terms of boundary y-values.

The aforementioned relation, Eq. (4), for determining the longshore sand transport gives the potential transport rate. If, for some portions of the beach, natural or man-made structures prevent the sand or shoreline from moving freely, these restrictions must be incorporated in the simulation model in terms of constraints.

In the following, the most commonly used boundary conditions are discussed. This discussion also demonstrates the vast difference in complexity between the explicit and the implicit calculation schemes. Similar discussions are given by Perlin and Dean (1978), and by Hanson and Kraus (1980).

5.6.1. Pinned beach

For the determination of locations along a beach to be used as model boundaries, it is often possible to find or assume the existence of a point that does not move appreciably in time. For the explicit scheme, this condition is formulated as

$$Q_1 = Q_2$$ (36)

or

$$Q_{N+1} = Q_N \tag{37}$$

depending on which end of the calculation grid is considered. In the implicit scheme (Eq. 32), a pinned beach at $i = 1$ is found to be given by

$$EE_1 = 1 \tag{38a}$$

$$FF_1 = 0 \tag{38b}$$

whereas the same condition at $i = N$ is given by

$$Q_{N+1} = FF_N / (1 - EE_N) \tag{39}$$

By inserting Eqs. (38) and (39), respectively, into Eq. (32), together with the appropriate i-values, relations identical to Eqs. (36) and (37) are obtained. Similar expressions can be derived for the reversed double sweep relation.

5.6.2. Groin

The presence of this type of structure is reflected either as a boundary condition or as a constraint, depending on the location. Placed on a boundary, the presence of a groin is reflected through a boundary condition, whereas located in the interior of the calculation grid, it is accounted for as a constraint. The formulation is the same for either case, although entering in different places in the numerical model.

The effect of a groin is formulated in terms of the transport rate passing the location of the groin, in proportion to the rate arriving at the adjacent grid point upstream of the groin. In GENESIS, sand can move past a groin by two mechanisms: either move around the groin (bypassing), or pass through the groin (transmission). The total fraction passing the groin out of the upstream transport rate, including both bypassing and permeabili-

ty, is formulated as

$$PB = PERM - PERM \; BYP + BYP \qquad (40)$$

where PERM denotes the groin permeability for sand transmitted through the groin $(0 \leqslant PERM \leqslant 1)$ and BYP denotes the bypassing factor $(0 \leqslant BYP \leqslant 1)$. Whereas in the model PERM is considered to be constant for a particular groin, BYP as mentioned above, depends on the actual wave conditions, and thus changes value every time step. In the explicit scheme, the boundary condition or constraint at a groin is formulated as

$$Q_G = PB \; Q_{G \pm 1} \qquad (41)$$

where the index "G" indicates the groin position, and the index "G ± 1" refers to the position next to the groin, on the upstream side $(= G + 1$ for negative transport direction).

In the implicit scheme, the presence of the groin must be formulated according to the longshore transport direction. For a negative transport direction, the expression becomes

$$EE_G = PB \qquad (42a)$$

$$FF_G = 0 \qquad (42b)$$

which inserted in Eq. (32) gives the desired relation

$$Q_G = EE_G \; Q_{G+1} + FF_G \qquad (43)$$

For a positive transport direction, the condition has to be formulated as

$$EE_G = 0 \qquad (44a)$$

$$FF_G = PB \; FF_{G-1} \; / \; (1 - PB \; EE_{G-1}) \qquad (44b)$$

and inserted in Eq. (32) results in

$$Q_G = PB\ FF_{G - 1} / (1 - PB\ EE_{G - 1}) =$$

$$= PB\ EE_{G - 1}\ Q_G + PB\ FF_{G - 1} =$$

$$= PB\ Q_{G - 1} \tag{45}$$

It is worthwile to note that if the passed amount is zero (PB = 0), Eqs. (42) and (44) become identical, and inserted into Eq. (32) leads to

$$Q_G = 0 \tag{46}$$

Also, if all of the material is passed (PB = 1), Eqs. (42) and (45), as boundary conditions, become identical to Eqs. (38) and (39) for the pinned beach case.

5.6.3. Seawall

In cases of chronic beach erosion, if the cause of the undesirable situation cannot be remedied, various types of coastal structures and/or non-structural methods (beach fill, vegetation) are used to protect selected portions of the threatened shore. If a line is identified, behind which the beach cannot be permitted to erode, and where non-structural measures are not feasible, seawalls (here used to include any type of non-erodible barrier along the shoreline) may be used to secure this line. It is, therefore, of great importance that the influence of these types of structures, on the shoreline evolution and the longshore sand transport rates, be included in the numerical model.

Although a large number of one-line model applications are presented in the literature, very few of these dicuss the influence of seawalls. Hashimoto et al. (1971) as well as Ozasa and Brampton (1980) set the longshore transport rate equal to zero where the the shoreline had retrerated to the seawall. Hanson and Kraus (1980) confined their procedure to adjusting the shoreline to the seawall position, whenever the transport calculation gave a location behind the structure.

To be complete, however, this measure must be followed by a correction of the calculated transport rates, in order to satisfy the sand continuity equation as expressed by Eq. (1). Such a method for preserving sand volume and transport direction is given in Hanson and Kraus (1985). Hanson and Kraus (1986) present calculated examples, together with complete program listings, for simulating the influence of a seawall in the one-line model.

A seawall imposes a constraint on the solution of the shoreline position, as the shoreline cannot move landward of the wall. The model calculates the longshore sand transport rates along the beach, based on the assupmtion that the calculated amounts of sand are available for transport. In cases where the associated shore-line change is in violation of the seawall constraint, the shore-line position, as well as the longshore sand transport rate, is corrected. As this is a very complex procedure, only a brief desription will be given here. For a complete description, see Hanson and Kraus (1986).

In the model, it is assumed that the shoreline can erode back to reach the seawall. At this stage, sand entering the area can pass through, but no further erosion is possible. Thus, local scour in front of the structure is not taken into account. Also, due to the properties of the one-line model, flanking is limited to shoreline retreat adjacent to the structure, and erosion behind the structure is not possible.

At a location where the shoreline position is in violation of the seawall constraint, the shoreline position is corrected according to (corrected values are denoted by a superscript asterisk)

$$y^* = ys \qquad (47)$$

where ys_i is the local seawall position. The corresponding correction of the transport rate is made as (in the explicit scheme)

$$Q^* = Q \frac{y - ys}{y - y'} \qquad (48)$$

The logic behind Eq. (48) is perhaps more clearly demonstrated after rearranging terms to give

$$\frac{Q}{y - y'} = \frac{Q^*}{y - ys} \qquad (49)$$

from where is is seen that whereas Q_i causes the shoreline to move from y_i to y_i', the corrected value, Q^*, moves the shoreline from y_i to ys_i.

6. STRUCTURE OF THE NUMERICAL MODEL

GENESIS can be thought of as consisting of two models - a wave model which calculates the breaking wave characteristics along-shore, and a transport model which calculates the longshore sand transport rate and the associated shoreline change. The wave model in GENESIS was developed to describe a wave field dominated by diffraction at structures. As such, it is based on the assumption of plane and parallel bottom contours. For open-coast calcula-tions without diffracting structures, it may desirable to use a more sophisticated wave model for bringing waves from deep to shallow water over an irregular bottom topography. At present, GENESIS is set up to communicate with a linear wave transformation model, RCPWAVE (Ebersole 1985; Ebersole, Cialone, and Prater 1985), giving the nearshore pre-breaking wave conditions. Subrou-tines in the wave model part of GENESIS then bring the nearshore waves to the breaking point.

The overall structure relating the shoreline simulation model, GENESIS, and the wave transformation model, RCPWAVE, is shown in Figure 8. Given the significant wave height, period, and angle at an offshore location (not necessarily in deep water), and the bottom topography, the transformation of the waves from the offshore model boundary to pre-specified nearshore locations can be calculated using the wave model.

6.1. Wave Model RCPWAVE

A two-dimensional grid is placed over a sea chart (see Figure 9), and the water depth in the center of each grid cell is deter-mined. It should be noted that the orientation of the x- and y-axes in the wave model are different from those in the shoreline model. Figure 9 shows the conventional axes orientation for a wave model. The output from the wave model is the wave height and direction (the wave period is assumed constant over the whole calculation grid) for each grid cell from offshore to the shore-line. The wave model is to provide GENESIS with nearshore wave data at each time step. As a typical time step for field use is 6 hours, and total simulation periods typically range from approxi-mately 2 to 20 years, it is obvious that it is usually not possible to run a wave refraction model once each time step.
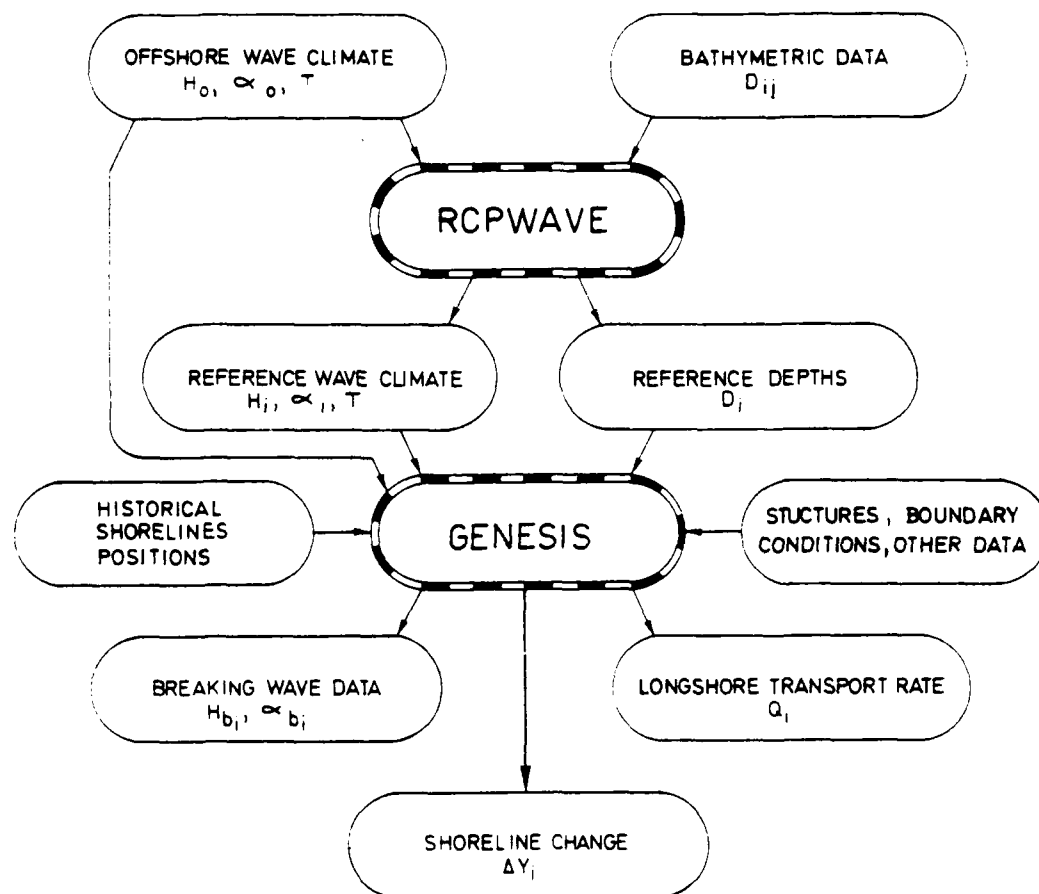
FIGURE 8.   Relation between the wave model, RCPWAVE, and the
shoreline change model GENESIS.

Instead,   another   technique   has been used. The offshore wave
data is divided into groups according to values of wave   direction
and period.   The number of groups is typically on the order of 50.
The wave model is then run once for each of these groups using   as
input   a unit wave height (1 m or 1 ft, depending on the system of
units used).   Because wave refraction and shoaling,   according   to
linear wave theory, are independent of wave height, this yields an
effective "transformation" coefficient comprised of the product of
refraction   and shoaling coefficients. This calculation also gives
the nearshore wave angles along   the   coast.   The   nearshore   wave
field   for   the   actual time series at, say, 6-hr intervals can be

easily obtained from GENESIS by first classifying the offshore
wave conditions according to the associated wave period and direc-
tion group. Then, the nearshore wave heights are obtained by
multiplying the "transformation" coefficients by the true offshore
wave height. The wave information is stored for points on a
pre-specified line, called the "reference" line (see Figure 9),
chosen to be close to, but still outside, the most seaward breaker
line location during the simulation period. The depths and wave
conditions along this line are stored in seperate files to serve
as input to GENESIS. In this procedure, it is assumed that no
structures that cause wave diffraction are present. Diffracting
structures are taken into account in GENESIS, through wave trans-
formation from the reference line to the wave breaking point.
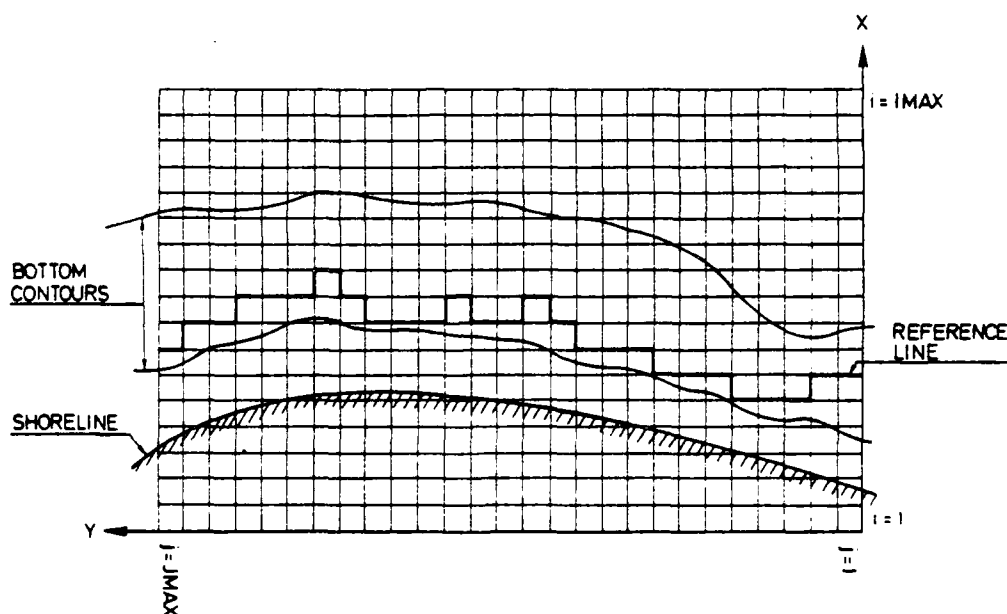
FIGURE 9.   Definition of coordinate system and reference
            line in RCPWAVE.

Although RCPWAVE is being employed at present to provide wave
input to GENESIS, any available wave transformation model may be
used to supply the necessary input wave data. Since RCPWAVE is a
totally independant program functioning as a service routine for
GENESIS, and as it is extensively described in Ebersole (1985) and
Ebersole et al. (1985), the model RCPWAVE will not be further
discussed here.

6.2. Structure of GENESIS

The main structure of GENESIS is shown in Figure 10, in which names of subroutines are enclosed by solid lines and names of data files by dashed lines. GENESIS is operated through interaction with data files. The files have been developed to allow representation of a large number and variety of coastal structure and shoreline configurations.

FIGURE 10. Main program structure of GENESIS.

The offshore as well as the nearshore (reference) wave parameters (height, period, and direction) are read from file WAVES. The corresponding nearshor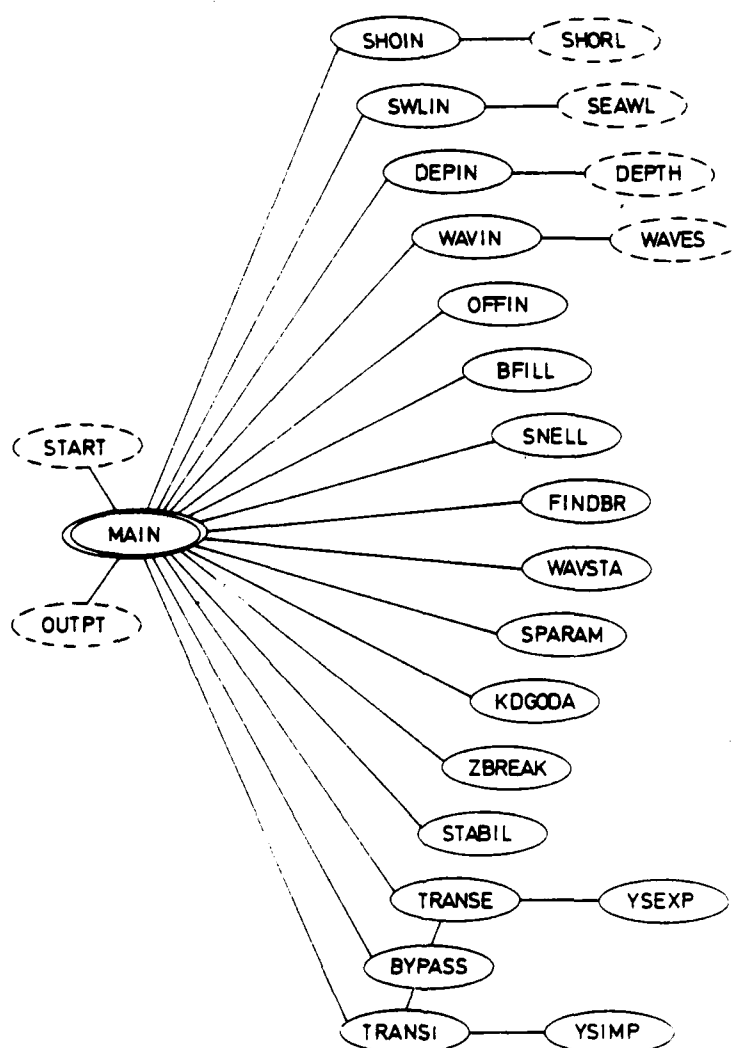e depths defining the reference line are stored in file DEPTH. The positions of seawalls, if any, and the initial position of the shoreline are read from files SEAWL and SHORL, respectively.

Other necessary information controlling the computation, such as the calculation increment in space and time, the total number of time loops and calculation grid cells, type of calculation scheme to be used (explicit or implicit), and also specifications of the properties and locations of the various structures at the site are located in the file START.

First, the breaking wave heights and directions are calculated along the coast, omitting wave diffraction as if the structures were not there. Then, the structures are introduced, treating each energy window separately. After the diffracted breaking wave heights and directions are determined, the associated longshore transport rates are calculated. This procedure is repeated for each of the energy windows, whereafter the transport rates are added to obtain the total rate as produced by all windows for each calculation element along the beach. Finally, the resulting shoreline changes are determined and, if the shoreline erodes past a seawall, corrected according to the seawall constraint.

The output from the model (calculated shoreline position and breaking wave height and direction along the shoreline) is placed in file OUTPT. Miscellaneous statistics, such as average wave heights and angles or the total longshore sand transport volumes along the coast, are placed in file STATS.

The principal functions of the individual subroutines will now be summarized (see next chapter for further information about the input data files). The subroutines are listed in order of their use. GENESIS first reads basic information from file START. Then GENESIS calls:

SHOIN: Reads the initial shoreline position from file SHORL.

SWLIN: Reads the seawall position from file SEAWL. If a particular application does not involve a seawall as specified in file START, this subroutine is not called.

DEPIN: Reads the depths along the reference line from file DEPTH, for which the wave model RCPWAVE calculates the nearshore wave conditions. From these points, the wave model in

GENESIS calculates the breaking wave conditions. (For infor-
mation on how to run GENESIS without involving RCPWAVE, see
the next chapter).

The three subroutines described above are called only once,
whereas the following subroutines are called at least once every
time loop.

WAVIN: Reads from file WAVES a single triplet of wave height,
direction, and period associated with an offshore pcint, and
one triplet for each of the nearshore points on the
reference line. If the wave model RCPWAVE is not used, only
the offshore values need to be specified.

OFFIN: One of the basic assumptions in the original one-line
formulation, is that the offshore contours move in parallel
to the shoreline (see Figure 11). If applied to the wave
model, this assumption may produce either numerical instabi-
lity or unrealistic wave refraction. Therefore, GENESIS is
calculating an offshore contour, representing the trend of
all bottom contours. The curvature of this line is obtained
by smoothing the shoreline to reflect the major features in
the shoreline, but to filter out possible abrupt variations.
This is done automatically once a month (model time scale)
in the model. Local wave refraction in GENESIS is performed
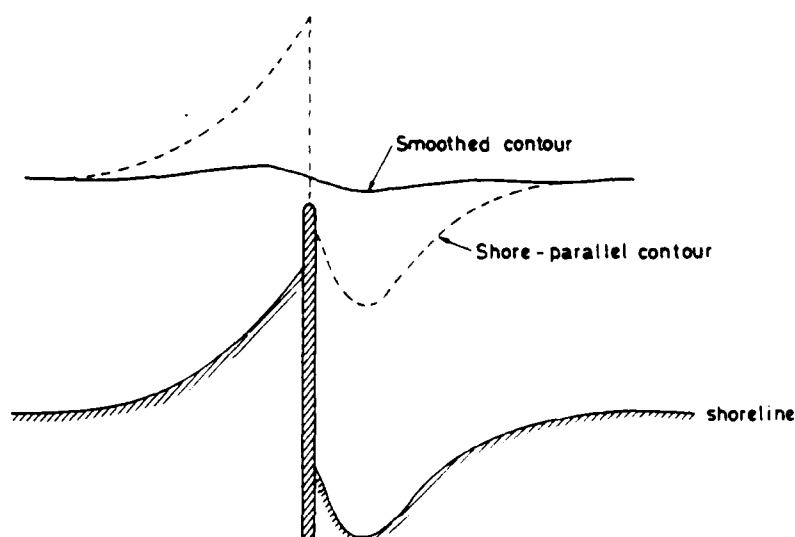using the orientation of this line.

FIGURE 11. Shore parallel versus smoothed bottom contours.

BFILL: Simulates the action of beach fill in terms of moving the beach. The locations, points of time, and amounts of fill material are specified in the START file by the user.

SNELL: Computes the wave number, wave group speed, refraction coefficient, and shoaling coefficient at a given depth for local application by GENESIS.

FINDBR: Calculates representative breaking wave heights and angles alongshore, neglecting the influence of diffraction.

WAVSTA: Gives selected wave statistics, such as mean breaking and undiffracted wave heights and angles at user-specified locations along the beach. These values are stored in file STATS.

SPARAM: Computes the value of the wave concentration parameter, S_{max}, at a diffracting tip using the offshore wave data. This quantity is needed for the diffraction calculation in subroutine KDGODA. If there are no diffracting structures, this subroutine is not called.

KDGODA: Calculates diffracted breaking wave parameters (H_b, Cg_b, α_b), as produced by the diffraction of random directional waves by structures. Calculating for one wave energy window at the time, the waves entering a particular window are diffracted by one or two structures. In the case of two structures, diffraction coefficients resulting from each of these are calculated separatly and then multiplied to get the total effect of that window. If there are no diffracting structures, this routine is not called.

ZBREAK: Modifies, through refraction, the breaking wave angles inside a shadow region calculated in KDGODA, with a correction for the local bottom contour orientation at each location. In cases with no diffracting structures, this subroutine is not called.

STABIL: Calculates the value of the stability parameter, for determining whether the explicit calculation scheme is unstable or the implicit scheme might produce unacceptable numerical errors.

TRANSE: Calculates the longshore sand transport rate associated with each energy window and, after all energy windows have been accounted for, computes the associated shoreline change, using the explicit calculation scheme.

TRANSI: Calculates longshore sand transport rates along the coast using the implicit calculation scheme.

BYPASS: Computes the percentage of sand bypassing a groin from the quantity leaving the adjacent calculation cell up-drift of the groin.

After the respective longshore transport rates are determined for each of the energy windows, they are added up. From this total transport, it is possible to calculate the associated shoreline change.

YSEXP: If the seawall constraint is violated, this subroutine corrects the longshore sand transport rates and shoreline positions in front of a seawall. Explicit calculation scheme.

YSIMP: Calculates the shoreline change along the coast for the implicit solution scheme. Recalculates, if necessary, longshore sand transport rates and the corresponding shore-line change, in accordance with the seawall constraint.

7.   DATA INPUT/OUTPUT

A numerical model of shoreline change can be a very powerful tool for predicting the change in beach planform under complex design and wave conditions. However, it is of great importance for the user to correcetly operate the model and interpret the results appropriately.   The user must be aware of all the underlaying assumptions and simplifications, as well as the general charac- teristics of the model. It is strongly recommended that the user of GENESIS should operate it for various simple conditions, to see how the model performs, before applying it to a prototype case.

Therefore, considerable effort was devoted to simplify the input interface. The data is read from five FORTRAN data files, in which the main parameter arrays (water depth; wave height, direction, and period; and shoreline and seawall positions) are separately held.   Other input values are collected in one data file (file START).   The following gives a brief description of these input files.

7.1.   File START

This data file contains a large number of input parameters which define the calculation environment. The most important of these parameters are:

Calculation scheme: The user can choose between an explicit and an implicit numerical solution scheme.

Number of calculation time loops.

Number of calculation grid cells.

Input/output units, which can be Metric or American Customary.

Time increment, dt.

Space increment alongshore, dx.

Average berm height.

Median sand grain size.

Groin locations and characteristics.

Detached breakwater locations and characteristics.

Beach fill characteristics: locations, durations, and amounts.

Longshore sand transport calibration parameters, K_1 and K_2.

7.2. Other Input Files

DEPTH: Contains the depths along the so called "reference" line. If the wave model RCPWAVE is not used and the wave parameters are given at one offshore point only, the model treats the bottom contours as parallel to the shoreline. In such a case, this data file is not needed (see next section).

WAVES: Once every time loop one set of wave data is read from this file. The structure of each set can be varied to a large extent, depending on the input wave description. If the wave transformation model RCPWAVE is used, each set will contain: the wave period (assumed constant over the full calculation area), the wave height and direction at one offshore location, and the height and direction for each wave model element along the reference line.

Instead of using an external wave model such as RCPWAVE to obtain the nearshore wave conditions, it is possible to use GENESIS for the same purpose. In this case, the wave set must only contain the wave height, period, and direction at the offshore location. This procedure implies that all bottom contour lines are parallel to the calculated offshore contour. In this case, the wave transformation calculations are performed directly from the offshore location to the breaker line. Thus, the reference depths are not used and the file DEPTH never called.

The number of sets in the wave file does not necessarily have to match the number of calculation time loops. The file is automatically rewound at the end of the file, and then read again from the beginning. A simple way to represent a wave climate which is constant in time, for test purposes, is to place only one set of wave data in this wave file.

SHORL: Holds the location of the initial shoreline.

SEAWL: Contains the location of either one continuous or a segmented seawall.

7.3.   Data Output

The most important data resulting from a calculation are stored in two data files, OUTPT and STATS. The former file contains information such as the alongshore distribution of the breaking wave heights and angles, and the longshore sand transport rate, all values given at the latest time step. This file also holds the initial and calculated shorelines at user specified time levels.

The file STATS holds the time averages, individual as well as combined, of breaking wave heights and angles alongshore, together with values on the net longshore sand transport volume per year at grid points along the shoreline.

8. CALCULATED EXAMPLES

Numerical models for simulating shoreline change can be used as an aid to understand the interaction between structures and sandy beaches in two fundamentally different ways. The effects of various parameters (e.g., groin orientation, beach fill location or breakwater configuration) can easily be isolated in a numerical model. It is therefore very educational to simulate simplified hypothetical cases and to interpret the results.

In situations, where the trend of shoreline change is very stable and when the structure configurations are expected to remain unchanged in the future, shoreline evolution can be forecast with reasonable confidence without the use of computers. However, in situations where the conditions controlling the shoreline change are either complex or vary during the forecast period, it becomes necessary to use numerical simulation models. Once such a model is successfully applied to a particular beach, future shoreline change under a large number of wave climates and structure configurations can be investigated, provided that the proper data are available.

To illustrate both the educational and applied uses of the model, two separate sets of of calculations were made. The first set involves a series of simple cases illustrating the influence of coastal structures on shoreline change. The second set is a limited effort to calibrate the model for a prototype case, concerning a beach fill made at Lorain, Lake Erie, Ohio.

8.1. Hypothetical Cases

For the sake of clarity, a number of separate simple cases will be presented. The offshore (d = 40 m) wave climate was held constant in time (H_0 = 1 m, α_0 = 30 degrees, and T = 5 seconds), and the initial shoreline was taken as a straight line. The structures used were: regular groins, T-groin, "dog-eared" groin, and seawalls. In addition, the effect of groin permeability is included. Calculation configuration common to all cases were:

Number of calulation grid cells = 70

Number of calulation time loops = 200

Time increment = 4 hr

Alongshore grid spacing = 25 m

Longshore transport calibration parameter K_1 = 0.5

Longshore transport calibration parameter K_2 = 0.4

8.1.1. Test case 1

Two impermeable groins, located in cells 20 and 40, respective-
ly, and extending 100 m and 150 m from the initial shoreline, were
modeled. The calculated result is shown in Figure 12, where the
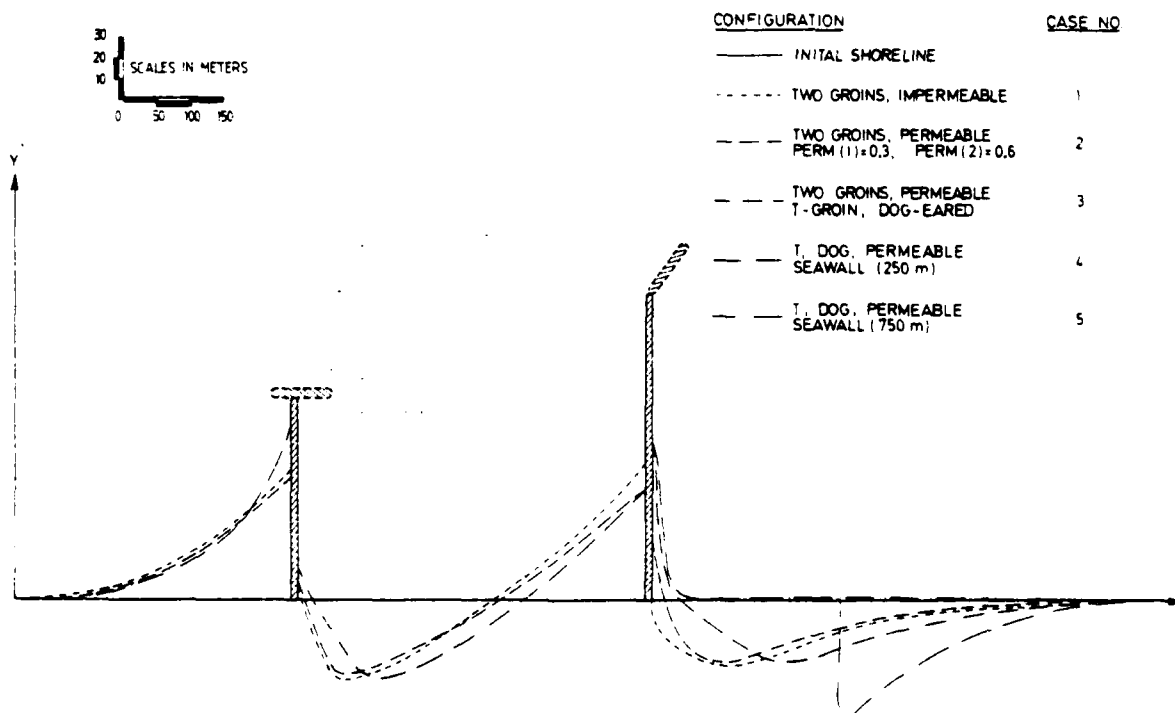line numbers refer to that of the respective test case.

FIGURE 12. Shoreline change for hypothetical
simulation examples.

The shoreline for this case behaves as expected: shoreline advance occurs on the up-drift sides of the groins and erosion on the down-drift sides. Due to wave diffraction, the locations of maximum erosion are not located immediatly downdrift of the respective groin, but at some distance away from the groin. The shoreline remains unchanged on the model boundaries, according to the boundary conditions used.

8.1.2.   Test case 2

Groin permeability, PERM, is defined as the amount of sand passing through the groin compared to the amount potentially available to be trapped, i.e., excluding the bypassed amount. This number, $0 \leqslant PERM \leqslant 1$, is user specified and treated as a property of the groin which remains constant in time. In order to study the effects of permeability, the two groins were made permeable to sand. The permeability coefficients used were 0.3 and 0.6 respectively, with the lower value associated with the left-hand groin.

The changes in shoreline positions are (Figure 12, line 2), compared to the previous Case 1, showing slightly decreased accretion on the up-drift sides of the groins as well as erosion on the down-drift sides, which is logical.

8.1.3.   Test case 3

The two permeable groins were replaced by a T-groin and a "dog-eared" groin, respectively (Figure 12, line 3). The T-bar was made 100 m long and the dog-ear was 56 m long, moving the groin tip 25 m further offshore.

On the up-drift side of the T-groin, wave diffraction produced dramatic shoreline advance adjacent to the groin. On the down-drift side the shoreline shape appears to have been moved alongshore with the diffracting tip. The shoreline movement on the up-drift side of the dog-eared groin is not affected by the dog-ear itself, but slightly by the T-groin. On the down-drift side of the dog-eared groin, the change is caused by the altered diffraction pattern. Close to the groin it seems like the change, as compared with case 2, is predominantly offshore and related to the offshore movement of the diffracting groin tip. Further

down-drift, however, the shoreline change seems to be more in the longshore direction and associated with the longshore movement of the groin tip.

8.1.4.   Test cases 4 and 5

As an attempt to prevent the severe erosion down-drift of the dog-eared groin, a seawall was placed on the original shoreline from that groin and 250 m down-drift (Figure 12, line 4). Although it serves its purpose for the portion of the beach it is set to protect, the problem area shifts further down-drift.

In case 5, the seawall was extended another 500 m. This measure holds the protected shoreline in place, but, as in the previous case, the erosion is not eliminated but moves even further down-drift.

Neither case, as expected, does effect the conditions up-drift of the dog-eared groin.

8.1.5.   Evaluation

A number of general conclusions can be drawn, based on the results of these simple calculations.

Shortly after the construction of a groin, sand is accumulated on the up-drift side, but approximately the same amount is lost from the down-drift side. Therefore, very little sand is gained to the total area. As the up-drift side is filled to its trapping capacity, sand start to bypass the groin. The sand deficit on the down-drift side is beginning to decrease and the beach is slowly recovering. At this stage, the total amount of sand in the vicinity of the structure is increasing. Thus, the length of a groin has to be determined in relation to the sand transport rate and the recovering (summer) season at the site it is set to protect. If the groin is too long, as in the examples above, significant sand bypassing never occurs and the beach is reformed rather than built up.

Detached breakwaters (T-groins, dog-eared groins) induce sand accumulation inside their shadow regions. This sand is, however, supplied from adjacent parts of the beach.

In order to be succesful, the two structural devices above should probably be combined with a beach nourishment plan, in order to build up the beach without causing down-drift erosion.

Seawalls are very powerful by means of preventing shoreline retreat. However, they only protect the coast within their own extent. In most cases, the erosion problem is transferred to the unprotected down-drift beach, if no other measures are taken.

8.2. Prototype Case

In 1977, three rubblemound detached breakwaters were consructed at Lakeview Park, Lake Erie, Lorain, Ohio. These were the first breakwaters in the United States, intended specifically to protect and stabilize a bathing beach, in this case an artificial beach fill (see Figure 13). The purpose of the fill was to protect the park and serve as a recreational beach at the same time. In addition to the breakwaters, the beach fill was held in place through the use of one groin on each side.

The shoreline and bottom contours were carefully monitored by the Corps of Engineers, both before and after the fill, providing excellent data for a numerical model simulation. Also, limited data are available on the statistical frequency of long-term directional distribution of wave heights and periods (Saville 1953; Resio and Vincent 1976). However, little informaton exists on the actual wave climate (height, period, and direction) between shore-line surveys. Thus, the wave series used in the model calibration/verification procedure had to be established for application of GENESIS.

As a test on the capability of GENESIS to reproduce prototype shoreline change, an attempt was made to simulate the shoreline change taking place during the first 24 days after the fill was completed. All necessary shoreline and structure configuration data were taken from survey charts (cf. Figure 13). The wave data immediately available was limited, only giving representative wave heights and periods from five different directions and their percentage distribution in time. It is therefore likely, that for a short-term simulation as made here, the actual mean wave climate could deviate significantly from the representative values.

Starting with the initial fill shoreline of 1 October, 1977, a series of simulations were carried out in order to reproduce the true shoreline of 24 October, 1977 (see Figure 14, line 2). In

addition to varying the calibration parameters K_1 and K_2, between the respective simulations, it was found necessary to assume that average deep water wave direction deviated 20° to the east from the representative values given by the input wave data. This calibration procedure suggested values of the two calibration parameters to be $K_1 = 0.3$ and $K_2 = 0.3$ (Figure 14, line 3).
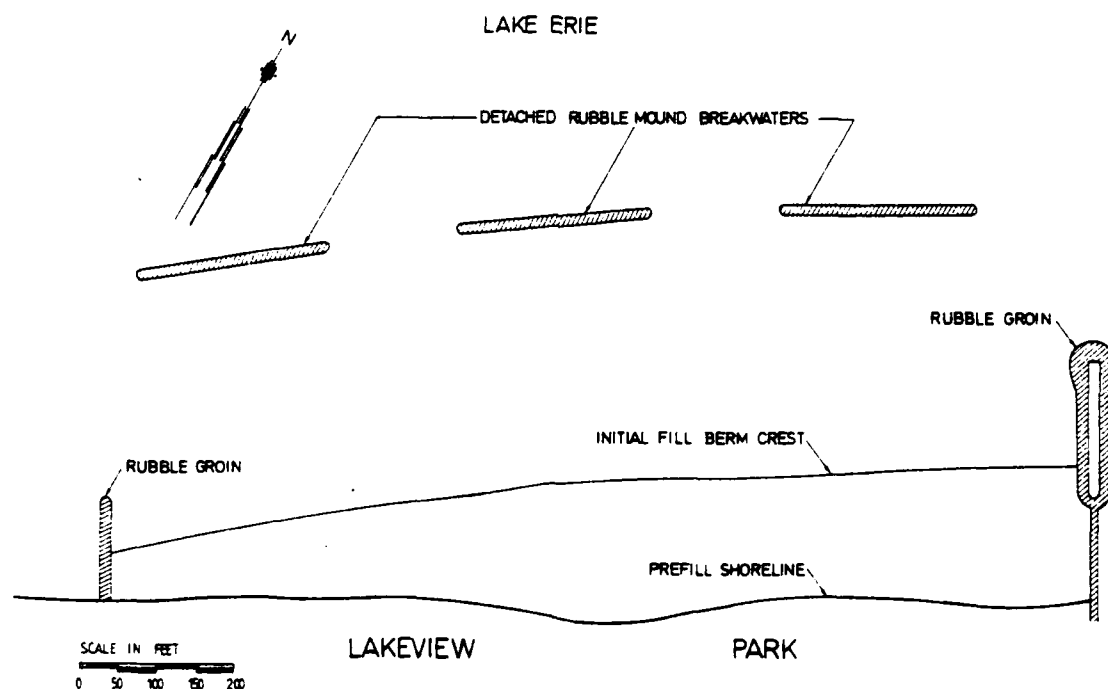

FIGURE 13. Shorelines and structure cofiguration at
Lakeview Park, Lorain, Ohio.

A comparison between the measured and the calculated shorelines of 24 October shows that the agreement, from a qualitative stand-point, is quite good. The model produces three well-developed salients (emerging tombolos) at the proper locations. However, the left-most calculated salient is somewhat too large whereas the

other two are too small. An explanation for these discrepancies could be the simplified description of the bathymetry in the area. Due to the limited available wave data, it was decided not to use the wave model RCPWAVE. Instead, all wave calculations were made within GENESIS, assuming bottom contours were parallel to the calculated representative offshore contour line.

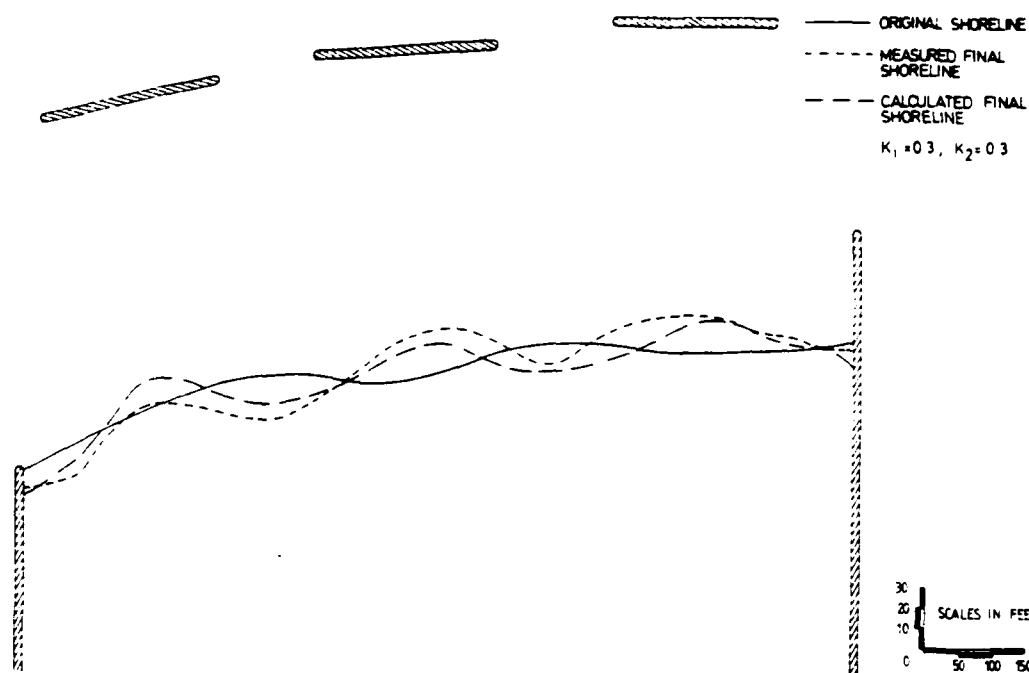FIGURE 14. Measured and calculated shorelines
at Lakeview Park.

In a beach fill project of this type, the volumetric changes can be as informative as the shape of the shoreline. In terms of this volumetric change, the computational results were very encouraging: the measured gain was 59,000 ft^3 and the calculated gain was 53,000 ft^3. Thus, the model accounted for 90 per cent of the volumetric change.
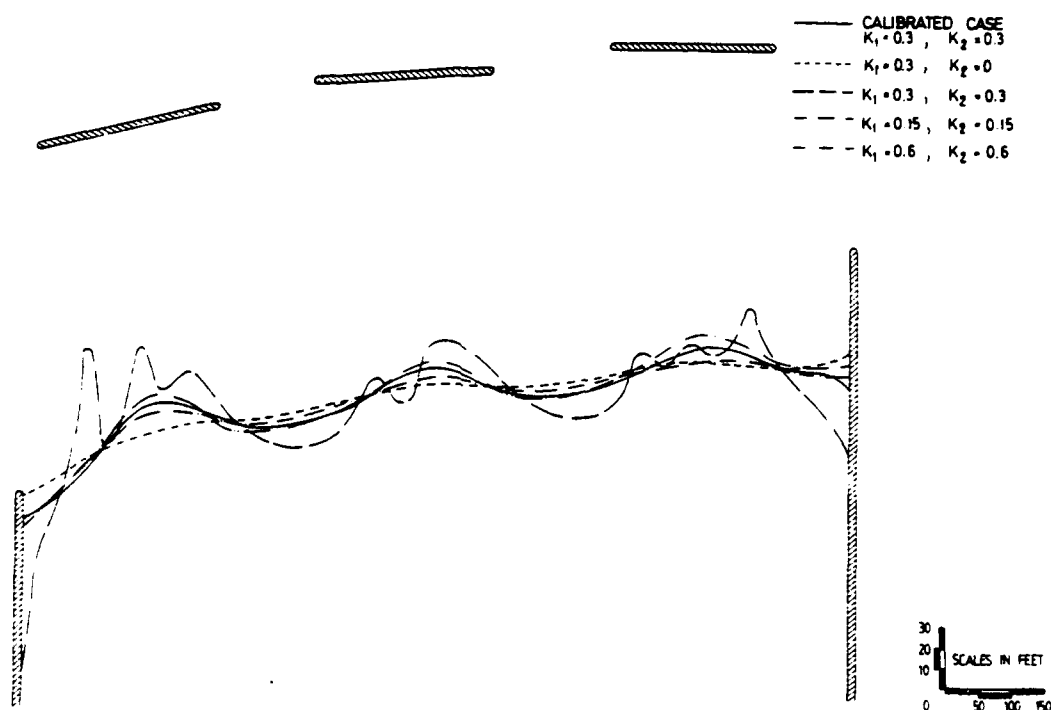
FIGURE 15. Result of sensitivity analysis for the
calibration parameters, K_1 and K_2.

The results of a sensitivity analysis on the calibration para-
meters K_1 and K_2 are shown in Figure 15. As a comparison, the
calculated calibration result is included (line 1). The first two
cases (lines 2 and 3) were simulated in order to demonstrate the
relative impact of the two terms in Eq. (4) on the calculated
shoreline. As expected, in Case 2, the first term alone will pro-
duce salients as a result of the change of the wave angles due to
diffraction, but they are much less pronounced than for the cali-
bration case. Setting the transport rate equal to solely the
second term in Eq. (4), (Case 3), the influence of the diffraction
is exaggerated. The strange shoreline configuration behind the two

outer detached breakwaters, showing three-peaked salients instead of the more common one- or two-peaked shapes, is believed to be a result of the waves having varying angles of approach. Thus, the two term in Eq. (4) act in opposite directions to form the shoreline shape: the K_1-term tends to flatten out the shoreline irregularities whereas the K_2-term promotes the development of salients (tombolos).

In Cases 4 and 5, the ratio K_1/K_2 is kept equal to 1 as in the calibration set-up, whereas the respective values of of the two calibration parameters are attenuated (Case 4) or amplified (line 5) by a factor of 2. The differences between the two curves are relatively small, implying that, for this case, the absolute values of K_1 and K_2 are of much lesser importance for the shoreline evolution than the ratio between the two terms. A probable explanation for this is that no structure (groin) with a significant trapping capacity is interrupting the longshore sand transport.

9. SUMMARY

Through the use of both laboratory and prototype data, a generalized 1-line model system (GENESIS) for simulating shoreline change was developed. The model incorporates a large number of physical features, such as sand bypassing and permeability of groins, wave refraction, shoaling, and multiple diffraction, as well as the effects of jetties, detached breakwaters, seawalls, and beach fills.

Much effort was spent on development of the data entry routines, allowing relatively easy configuration and modification of the shoreline, wave characteristics, and coastal structures. The goal was to enable the user to apply the model to almost any laboratory or prototype open-coast situation with a minimum amount of work.

Calculated examples show that GENESIS reasonably describes a variety of hypothetical situations, and that it is possible to use the modeling system for preliminary prototype design. Still, much model development remains to be done, of which main improvements, already in progress, are representation of wave transmission through detached breakwaters and capability to describe sources and sinks of sand along the shore.

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